SECTION 3.0

GEOTECHNICAL ANALYSIS

3.0 GEOTECHNICAL ANALYSIS

This chapter discusses the results of the geologic and hydrogeologic investigations that were incorporated into the various geotechnical analyses required to support the currently proposed prescriptive design for the Gregory Canyon Landfill. Specifically, this section of the report provides the results of static and dynamic slope stability analyses, a discussion of liquefaction analyses for the alluvial material at the toe of the landfill, an assessment of rockfall and debris flows, settlement analysis over a period of 60 years, and model results for the generation of leachate by the landfill.

3.1 LINER SYSTEM DESIGN

The currently proposed landfill design includes the excavation of alluvial and colluvial deposits, and sufficient bedrock materials to create a subgrade at a minimum of 5 feet above the highest anticipated groundwater elevation. As shown on Figure 3-1, the bottom area of the footprint will be graded to drain northerly at a minimum gradient of three percent. The interior side slopes will be cut at a gradient no steeper than 1:1 (horizontal to vertical). The elevations of the finished bottom subgrade or floor area for the refuse footprint range from between approximately 380 feet amsl at the northwestern corner to about 750 feet amsl in the southern portion of the floor area.

The composite liner system proposed for the GCLF will include a GCL (a geocomposite clay liner) sandwiched between an upper 80-mil high density polyethylene (HDPE) geomembrane and a lower 60-mil HDPE geomembrane liner. On the floor areas, the encapsulated GCL will overlie a nine-inch drainage (leak detection) layer over a 60-mil HDPE geomembrane liner, all of which will overlie a compacted low permeability soil layer. As shown on Figure 3-2, from top to bottom, the components for the proposed composite liner system include the following:

- 24-inch-thick protective soil cover
- 12 oz. geotextile (on floor areas only)
- 12-inch thick LCRS gravel layer (on floor areas only)
- 16 oz. cushion geotextile
- 80-mil HDPE geomembrane, textured as required
- Geocomposite Clay Liner (GCL)
- 60 mil HDPE geomembrane textured both sides
- 16 oz. geotextile
- 9-inch minimum thickness gravel or equivalent drainage layer, with a collection pipe (on floor areas only)
- 16 oz. geotextile (on floor areas only)
- 60-mil HDPE geomembrane, textured both sides (on floor areas only)
- 24-inch-thick low permeability soil layer
- 12 oz. geotextile (on floor areas only)
- 12-inch-thick subdrain gravel layer (on floor areas only)

The subdrain system, though not expected to contain groundwater, has been included in the landfill design to collect and control groundwater, if it intersects the subgrade excavation along the bottom. The subdrain system will be placed beneath the composite liner and will consist of a one-foot-thick gravel blanket and gravel-filled trenches with slotted collector pipes in the bottom areas. The subdrain system is a redundant system in which the permeable gravel pack and the pipe can both convey over a million gallons of groundwater per day by gravity flow to the mouth of the canyon where it will be captured in a collection tank. On the side slopes, if a groundwater seep is identified, depending on the measured flows, either a geonet or trench-type collector chimney drain will be constructed, for lower flow seeps, a geonet strip collector will be used . The collector will be placed from the seep to the next lower bench into a section of slotted pipe surrounded with gravel and wrapped in geotextile. The slotted pipe will transition to solid pipe gravity flowing to the bottom subdrain system. For higher flow seeps, a trench collector type chimney drain will be constructed by first cutting a trench into the side slope from the next lower bench up to the seep. The trench will be filled with gravel and wrapped with geotextile. A perforated pipe can also be added for additional flow capacity. The trench size will be dictated by flow rates. The trench collector will connect at the bench and eventually to the bottom subdrain system similar to the geonet collector.

In floor areas, a 12 ounce geotextile layer separates the 12-inch thick subdrain gravel from the 24-inch low-permeability soil layer. The low-permeability soil layer will be compacted to a minimum of 90% relative compaction (per ASTM D1557) and yield a maximum hydraulic conductivity of $1x10^{-7}$ cm/sec.

A 60-mil HDPE geomembrane, textured on both sides, will be placed above the low-permeability soil and provides both a low permeability and high resistance to chemicals. The complementary physical and hydraulic properties associated with the combination of a geomembrane and low permeability soil result in a very low potential for leakage, many times lower than that of either layer alone.

In the floor areas a 16 ounce geotextile overlies a drainage (leak detection) layer, which consists of a nine-inch minimum thickness of gravel or equivalent with a collection pipe to create the drainage (leak detection) layer. The drainage layer will then be overlain by a 16 ounce geotextile and a 60- mil HDPE geomembrane, textured on both sides. The texturing improves the interface shear strength between the geomembrane and the geotextile below and the GCL above. The geomembrane will be constructed so that its liner seams will be offset from the seams in the lowermost geomembrane (below the drainage layer).

A geocomposite liner (GCL) will be placed above the geomembrane. The GCL is a commercially manufactured liner material containing sodium bentonite. It can be attached directly to the geomembrane or be sandwiched between two geotextiles. The hydraulic conductivity of a typical GCL is on the order of $5x10^{-9}$ cm/sec. The GCL is dry when it is placed and therefore, the bentonite in the GCL has the ability to swell and self-heal and can deform and stretch significantly without losing its hydraulic integrity.

An 80-mil HDPE geomemembrane, textured on both sides in the floor areas, will be placed above the GCL. On the slopes, the upper (80 mil) HDPE geomembrane will consist of a single-side textured product placed with the textured side down. As with the HDPE geomembrane beneath the GCL, this geomembrane provides additional protection to the low-permeability GCL by its high resistance to chemicals and lower permeability. The thicker 80-mil HDPE geomembrane liner also provides added resistance to punctures or damage by surface construction activities following its placement. The texturing improves the interface shear strength between the geomembrane and the geotextile above and the GCL below. This uppermost geomembrane will be constructed so that its liner seams will be offset from the seams in the underlying geomembrane (below the GCL). A brief discussion of the performance advantage of the GCL sandwich design is provided in a letter to Gregory Canyon Ltd. (April 2004).

A Leachate Collection and Removal System (LCRS) will be installed over the 80-mil geomembrane liner to collect and convey leachate that may be generated within the refuse prism. In the floor areas, the system will consist of a one-foot thick gravel layer and HDPE pipe over the entire bottom area, while the slope areas will consist of gravel pipe and gravel collectors wrapped with a geotextile fabric placed on the interior benches. The bottom and slope collectors will be interconnected to convey leachate by gravity flow to the mouth of the canyon to be discharged into two double-walled collection tanks. Though the LCRS system has been designed to prevent clogging of gravel and the pipe layer, in the event of system back-up, clean-outs will be available that can be periodically flushed. In the event of pipe failure, the surrounding gravel layer also has the capacity to convey the leachate to an adjacent pipe or outlet system.

A protective soil cover consisting of a minimum two-foot thick soil layer will be placed over the entire liner and/or LCRS. A 12 ounce geotextile will separate the protective cover from the underlying LCRS/liner.

3.2 SUPPORTING FACILITIES/LANDFILL COMPONENTS

The following section provides a description of additional landfill support facilities and components that require geotechnical analyses or evaluation including the ancillary facilities area (liquefaction analysis), borrow/stockpile areas (slope stability), the access roads and bridge (construction considerations).

3.2.1 Ancillary Facilities

The ancillary facilities area will be located at the toe of the landfill and cover an area of about 12 acres. It will include the fee booth and scales, the administrative office building (located adjacent to the fee booths), and an approximately 7,000 square foot maintenance building. A diesel storage tank within a concrete containment wall will be located on the south side of the maintenance building for refueling of equipment. A portable emergency showerhead designated to contain rinse water will also be provided outside the maintenance building.

A recyclable drop-off area is proposed on the east side of the maintenance building with bins for drop-off of source separated recyclable material, such as newsprint, white paper, tin, aluminum, and glass. White goods will also be accepted and stored near the recycled bins area. Although hazardous materials will be prohibited at Gregory Canyon Landfill, a hazardous materials storage area, located in the southeastern portion of the ancillary facilities area, will be maintained for use if such materials are found in loads coming to the landfill.

Two 10,000-gallon leachate holding tanks and one 10,000-gallon subdrain water tank will be located in the southwestern corner of the ancillary facilities area. A 20,000 gallon water tank will also be located just north of the paved area. The water tank will be supplied from on-site groundwater wells.

3.2.2 Borrow/Stockpile Areas

Approximately 87 acres of borrow/stockpile area will be provided at two locations to the west of the proposed landfill footprint (Figure 2-7). Borrow/Stockpile Area A, which is about 22 acres in size, is located west of the landfill footprint adjacent to the western property boundary. The maximum height of Borrow/Stockpile A ranges from about 320 to 500 feet amsl. Borrow/Stockpile Area B, which is about 65 acres in size, is located immediately to the west of the southern portion of the landfill footprint. The maximum height of the Borrow/Stockpile Area B ranges from about 940 to 1,020 feet amsl.

The borrow/stockpile areas will be used to store and/or excavate material that will be needed in the daily operation of the landfill. During the initial excavation of Phase I of the refuse footprint, a portion of the excavated material will be used for the engineered fill necessary to construct the ancillary facilities area. The remainder of the material will be stockpiled in the landfill footprint or Borrow/Stockpile Area A (Figure 2-7). Borrow/ Stockpile Area A will be used for stockpiling only during the initial construction after which the area will be closed and revegetated with native plant species. Area A will not be used again for about 25 years at which time material will be used from Area A for cover. In the interim, material will be stockpiled within the footprint or in Borrow/Stockpile Area B.

A geophysical study of the borrow areas was conducted to evaluate the rippability of the soil for use by the landfill (GLA, 1998). On the basis of these studies, Borrow Area A is considered rippable to an average depth of 80 feet, and Borrow Area B is considered rippable to an average depth of 75 feet. These rippable depths suggest that approximately 1.3 million cubic yards of material can be excavated from Borrow Area A with conventional earth-moving equipment. Borrow Area B is to be excavated to a depth of about 150 feet and is expected to yield up to 3.2 million cubic yards of material. Though the footprint for the maximum depths is relatively small, deep excavation may require the use of explosives.

Proper drainage control will be maintained in the borrow/stockpile areas. Surface water control features will include grading of the flatter deck areas to promote lateral runoff of precipitation into drainage control facilities such as downdrains and bench drains on the slopes. Surface waters will be collected and conveyed from the borrow/stockpile areas and discharged into the existing natural drainage courses. Erosion control measures such as desilting basins, sandbags, straw matting and/or rip-rap will be utilized to reduce downstream siltation potential. Discharge rates will be equal to or less than natural flow conditions.

Borrow/Stockpile Area B will drain to the southwest into a natural drainage course. At the western end of the Borrow/Stockpile Area B, a desilting basin will be constructed to minimize the flow of silt from the borrow/stockpile area. The desilting basin will be designed to accommodate the soil loss from the borrow/stockpile areas. The drainage course for Borrow/Stockpile Area A runs northwesterly. The drainage control facilities will direct the surface runoff into the existing streams.

Interim drainage and erosion control features (e.g., silt fences) will be constructed for the borrow/stockpile areas, as necessary. Construction and operation of all drainage facilities will adhere to the BMPs developed as part of the Storm Water Pollution Prevention Program Plan (SWPPP) in compliance with State and Federal regulations under the National Pollutant Discharge Elimination System (NPDES) program. The pre-developed drainage condition of the area will be reconstructed as closely as possible once operations are discontinued in each of the borrow/stockpile areas. Exposed areas will be revegetated with native plant species to prevent erosion.

3.2.3 Access Roads/Bridge

The proposed access road from SR 76, which will extend through the abandoned Lucio dairy to the ancillary facilities area, will be two to three lanes and will include a bridge over the San Luis Rey River. The access road from SR 76 to the bridge will be about 910 linear feet and will be 32 feet wide, with two twelve-foot travel lanes and a four-foot shoulder on each side. The access road from the bridge into the ancillary facilities area will be about 985 linear feet and will be 36 feet wide, with three lanes (two travel lanes and a center lane) and a four foot shoulder on each side.

The bridge will be approximately 680 feet long, with five 7-foot diameter supporting piers, which will form the base of the structure. The 35.5-foot wide bridge will have two travel lanes and will maintain a 17.5-foot clearance between the proposed finished channel bottom and the underside of the bridge. The side slopes at the bridge abutments will be graded to a maximum 2.5:1 slope gradient and will be stabilized with rip-rap to prevent erosion.

3.3 STABILITY ANALYSES

The following sections provide the results of geotechnical slope stability analyses that were performed for the design of the proposed landfill and key appurtenant facilities of the project, described above. The geotechnical analyses include static stability of the cut slopes and the refuse prism, the dynamic stability of the refuse prism and the potential for rockfall to occur from the surrounding highlands and impact the landfill.

3.3.1 Stability of Cut Slopes

The three most common types of cut-slope failures are block-slip failures, wedge-slip failures, and circular failures. Block-slip failures are most common in slopes that are underlain by bedrock with distinctive partings (e.g., fractures) that dip in the same direction but at a shallower angle than the cut. Wedge-slip failures occur when the bedrock has two or more partings (e.g., a weathered dike and a joint) with orientations such that their line of intersection dips at a shallow angle in the direction of the cut. Finally, circular failures develop where the substrate is loosely consolidated and comparatively homogeneous. Rocks exposed in Gregory Canyon are compact and cohesive, even when weathered, so a circular failure of the cut slopes is unlikely and will not be considered below.

A kinematic analysis of cut slopes for the proposed landfill development was conducted and utilized the fracture data from 424 measurements in bedrock outcrop and bore hole imaging probe data generated from 12 borings (GLA, 1998). The kinematic analysis showed that large-scale block-slip movement and wedge-failure are not likely given the geometry of the dominant directions of discontinuity in Gregory Canyon identified by the geologic investigations. Mapping should be performed and this conclusion reevaluated as the excavation proceeds. It is also possible that small-scale, localized block falls may occur when fractures daylight the cut or where a higher density of fractures are encountered during excavation.

A study conducted by Woodward-Clyde Consultants (1995) concluded that 2:1 slopes on the east-facing slopes adjacent to the aqueduct are appropriate with a factor of safety of at least 1.5 under static conditions. In response to concern about the stability of the first San Diego Aqueduct during an earthquake event, GLA also performed a pseudo-static analysis of the proposed east-facing cut slopes (adjacent to the aqueduct). Static analysis of modeled wedges indicates a factor of safety of 5.9. This means that the forces resisting movement are approximately six times greater than the forces causing movement. When subjected to ground acceleration associated with the Maximum Credible Earthquake (0.4g), the factor of safety also exceeds the prescriptive 1.5 dynamic factor of safety for all landfill foundation and final fill slopes required by 27 CCR.

<u>Borrow/Stockpile Areas</u>. Within the borrow/stockpile areas, block-slip failures, wedge-slip failures, and circular failures were all assessed (GLA, 1998). Based on the kinetic considerations and structural features of the rocks exposed at Gregory Canyon, it was

concluded that block failures, wedge-slip failures and circular failures of cut slopes with 2:1 (H:V) slopes are not likely. As a result, cut slopes for borrow excavations, will be made at a maximum gradient of 2:1 (H:V).

3.3.2 Stability of the Stockpile Slopes

Stockpiles will be placed in the borrow areas at a maximum slope gradient of 3:1 (H:V) and a maximum height of no more than 300 feet. The computer program SLOPE/W was used to analyze the static stability for two cross-sections through these slopes (Figure 2-7). Based on the nature of the materials anticipated to be placed in the stockpiles, a unit weight of 120 pcf, a friction angle of 32° and cohesion of 250 psf were considered reasonable and were used in the slope stability analysis. Results of the analysis (Figures 3-3A, and 3-3B) indicated a calculated minimum static factor of safety of 1.9. Based on this evaluation, the stockpile slopes were considered to have adequate stability.

3.3.3 Stability Analysis of the Refuse Slopes

The bottom grade for the liner, the liner system details, and the finished grades were provided and are presented on Figures 3-1 and 3-2 (BAS, 2003). The interface strengths for each of the liner components can be measured and expressed in terms of standard Mohr-Coulomb strength envelopes (stated in terms of an angle of internal friction and cohesion or adhesion). For the purpose of performing slope stability calculations discussed herein, only the weakest or most critical elements of the liner system have been used. For analysis, the critical elements of the refuse slopes and liners are the refuse fill and the interface between the non-woven geotextile and the HDPE membrane. The parameters used in the analyses and derived primarily from published data (e.g., Serrot International) were:

Material	Unit Weight	Friction Angle	Cohesion
Refuse Fill	80 pcf	30°	200 psf
Smooth HDPE/Geotextile	NA	8°	0 psf
Textured HDPE/Geotextile	NA	14°	0 psf

Geometry. Cross section A-A' shows the final grade profile of the GCLF along the approximate center of Gregory Canyon and shows the proposed landfill configuration and incorporates the most critical section with regard to slope stability for the site (Figure 3-4).

Static Analyses

Slope stability analysis was conducted using the computer program SLOPE/W (Geo-Slope, 1995). Analytical methods available in the program include Bishop method for circular failure modes, and Spencer and Morgenstern and Price methods for general failure modes including block and non-circular failure surfaces. The analysis computes the factor of safety against failure using limit equilibrium procedures. Input parameters required for the analysis include the geometry of the slope, the unit weights and shear strengths (angle of internal friction and cohesion) of the various materials and interfaces in the slope and pore water pressure conditions, if appropriate.

Figures 3-5 shows the critical failure plane for section A-A, and the static factor of safety for this configuration is calculated to be greater than 1.50.

Dynamic Analysis

Current practice for evaluating dynamic slope stability under seismic loading conditions involves two steps: (1) pseudo-static analysis; and (2) more detailed analysis, if step (1) indicates a factor of safety less than 1.5.

In the pseudostatic analysis, the dynamic inertial forces induced by the design earthquake are represented by a constant, equivalent static horizontal force, determined by multiplying the weight of the slope by a seismic coefficient (k). This is a simplified representation of a complex dynamic condition (hence the term 'pseudostatic'). It is customary in the practice, to use a seismic coefficient (k) of 0.15 for slope stability analysis.

If the pseudostatic slope stability analysis calculates a factor of safety of 1.5 or more, the slope is considered to have adequate stability under seismic loading conditions. Otherwise, Title 27 requires that further analysis be done, as step (2) to demonstrate that the proposed design will be functional during the design earthquake for the project (the Maximum Credible Earthquake [MCE] for the GCLF). This additional analysis may include determination of seismic-induced permanent displacement.

Pseudostatic analyses for section A-A' indicates a factor of safety of 0.85 with a seismic coefficient of 0.15 (Figure 3-6). Since the calculated factor of safety is less than 1.5, additional pseudostatic analysis was completed and included estimation of the seismic-induced permanent displacement due to the MCE.

In evaluating seismic displacement, the procedure by Bray and Rathje (1998) was used to estimate seismic-induced permanent displacement during the MCE. The procedure is based on the one described by Newmark (1965) for determining displacement of a rigid block resting on a sliding plane subjected to earthquake-type motions. The procedure first requires determination of "yield acceleration" (i.e., the value of seismic coefficient (k_y) that yields a factor of safety of 1.0 in conventional slope stability analysis). This procedure is then based on the premise that the sliding block will undergo displacement only during periods when the maximum ground acceleration (k_{max}) exceeds the yield acceleration (k_y) for the sliding block. As a result, no displacements occur when k_y is greater than k_{max} (i.e., $k_y/k_{max} > 1$). Bray and Rathje refined the procedure for solid waste landfills to incorporate the dynamic response characteristics of the waste fill, and the intensity, frequency content, and duration of ground motion. The Bray and Rathje procedure yields results that are consistent with the observed performance of landfills during recent earthquakes.

The yield acceleration for section A-A is calculated to be 0.10g (Figure 3-7). Based on this yield acceleration, seismic-induced permanent displacement of the landfill slopes was estimated for the peak horizontal acceleration of 0.40g for the MCE event at the site (See Section 1.4.4). The following parameters were used:

- Slope Height - 300 feet
- Average Shear Wave Velocity of Refuse Fill – 1,200 feet/second (Bray and Rathje, 1998)
- MCE - M7.1 event on the Elsinore Fault-Julian Segment a distance of 6 miles from the site
- MCE Site Acceleration – 0.40g
- Mean Period of Shaking – 0.50 seconds (Bray et al, 1998)
- Significant Duration of MCE – 16 seconds (Bray et al, 1998)

Based on the analysis, the displacement calculated to occur to the total refuse prism and liner is approximately 0.1 inches for the prescriptive configuration. This is less than the commonly accepted maximum displacements for liner systems of 6 inches to 12 inches (Seed and Bonaparte, 1992). Attachment 5 provides the calculation used to determine the seismic-induced permanent displacement for the GCLF base liner along cross-section A-A'.

3.3.4 Stability of the Final Cover

In addition to an evaluation of the cut slopes, and refuse prism, stability analyses were performed for the proposed final cover system for closure of the GCLF. The proposed final cover design assumes a prescriptive low-permeability final cover in accordance with CCR 27. It consists of a two-foot foundation soil layer, a synthetic barrier layer, and a two-foot thick vegetative soil layer. The vegetative soil layer was assumed to consist of on-site soils that are silty sand to sandy silt and compacted to a minimum relative compaction of 90 percent. The barrier layer was assumed to consist of a 60-mil thick Linear Low-Density Polyethylene (LLDPE) geomembrane, textured on both sides. The foundation layer was assumed to consist of compacted random soil. The proposed final grading plan will have an overall slope gradient of 4:1 (horizontal: vertical) including roads and benches at approximately 40-foot vertical intervals and a gradient between benches of 3:1 (horizontal: vertical).

For the slope stability analysis, the interface between the LLDPE geomembrane and the overlying vegetative soil cover was considered the critical surface. The following parameters were considered appropriate and used in the analysis:

Thickness of vegetative soil layer	2 feet
Total density of soil in the vegetative layer	100 pcf
Angle of internal friction at the interface between soil and LLDPE geomembrane	27 degrees
Maximum ground acceleration for the postulated maximum credible earthquake (MCE) at the site	0.40g

The slope stability analysis was conducted considering the proposed final cover as a semi-infinite slope with a gradient of 3:1 (horizontal: vertical). For the design parameters listed above, the analysis indicated a static factor of safety of 1.53 if the tensile strength of the geomembrane is ignored, and 1.69 when considering the tensile strength of the LLDPE.

Since the pseudo-static factor of safety was less than 1.5 (see Attachment 5), an additional analysis was made to estimate the seismic induced permanent displacement during the postulated seismic exposure of the site using the procedure described by Makdisi and Seed (1978). The procedure first requires calculation of yield acceleration (k_y), the acceleration value for which a pseudo-static analysis yields a factor of safety of 1.0. K_y was evaluated and found to be equal to 0.185g. The ratio k_y/k_{max}, where k_{max} is the maximum ground acceleration at the site (0.40g), was then calculated. The value of the estimated permanent displacement was then read from a chart developed by Makdisi and Seed normalized for the period of the waste and related to the magnitude of the earthquake event. Using this procedure, the calculated seismic-induced permanent displacement for the final cover during the postulated maximum credible earthquake at the landfill ranges from 1.7 to 5.1 inches (Attachment 5) depending on the thickness of the waste prism. Using the methods of Bray and Rathje (1998), the seismic displacement under the loading of the MCE ranges from 0.5 to 3.7 inches, depending on the waste thickness (Attachment 5). These estimated displacements are less than the commonly acceptable range of seismic displacements of 6 to 12 inches and would not be expected to inhibit the functional integrity of the cover. Damage should be evident during post-earthquake inspection, and can be easily and quickly repaired as a part of post-earthquake maintenance. The seismic-induced permanent displacement calculations are provided in Attachment 5.

3.4 LIQUEFACTION SUSCEPTIBILITY

To aid in assessing the liquefaction potential of the alluvium underlying the site, the alluvial wedge was drilled, sampled and field tested to a depth of 50 feet at four different locations (Figure 1-2). Laboratory tests were then completed on representative samples in order to characterize the grain-size distribution of the alluvial soils (GLA, 1998). On the basis of the testing, it is concluded that the soils are predominantly silty sands and clayey sands with 14% to 45% silt and clay at the tested locations. A detailed discussion of analytical procedures employed in the liquefaction analysis is provided in GLA's geotechnical report (GLA, 1998). The following table summarizes the SPT data from Gregory Canyon, the cyclic stress ratios calculated for each SPT location, and the calculated factors of safety.

Boring Number	Depth in feet	Material	Percent fines	Depth to water table (ft)	C_d First cyclic-stress ratio	C_l Second cyclic-stress ratio	Factor of safety
Boring 1	20	SM	17%	19	0.199	0.300	1.50
	30	SM	19%	19	0.228	0.700	3.07
	40	SM		19	0.230	0.672	2.93
Boring 2	30	SP/SM	14%	25	0.204	0.280	1.37
	40	SM		25	0.211	0.669	3.16
Boring 3	25	SM-SC	45%	24	0.195	0.400	2.06
	35	SM-SC		24	0.211	0.671	3.18
	45	SM		24	0.208	0.357	1.72
	55	SM		24	0.188	0.630	3.34
Boring 4	35	SM-SC	20%	26	0.206	0.269	1.30
	45	SM-SC		26	0.202	0.651	3.22

Notes: C_d – Ratio of cyclic stress induced by the earthquake to effective normal stress
C_l - Ratio of cyclic stress required to induce liquefaction during the earthquake to effective normal stress.

The lowest calculated factor of safety is 1.30 (in Boring 4), and most other values are considerably higher. For liquefaction hazards, factors of safety ranging from 1.25 to 1.5 are generally considered acceptable and within the standard of practice (SCEC, 1999). We conclude that under existing conditions the liquefaction susceptibility of the alluvial wedge at Gregory Canyon is low, and not a significant impact to the project.

3.5 ROCKFALL AND DEBRIS FLOW ASSESSMENT

Rockfall

Rockfalls are abrupt movements of independent rock blocks detached from steep slopes. Falling rocks can reach the base of a slope by free-falling, bouncing, rolling down the slope surface, or by some combination of the above. Such independent rockfall movements differ from sliding failures that form on a slip surface of the rock slope in that they are much more sudden, and because falling rocks may possess a high degree of kinetic energy. If the failure involves a large volume of rocks, debris will cover a very wide area at the base of the slope.

Potential for rockfall was assessed using trajectory analysis of bounding rock fragments under the assumption of ideal elastic behavior for rock slopes. A representative cross-section of the Gregory Canyon from the summit of Gregory Mountain to the thalweg of Gregory Canyon was considered in this evaluation (GLA, 1998). The analyzed profile includes some of the steepest and longest slopes, and is relatively free of transverse features (e.g., ravines) that could otherwise act as catchment basins for rolling fragments. In considering the rockfall hazard, three scenarios were considered: (a) complete dampening of the kinetic energy once the particle landed in the comparatively soft refuse fill; (b) some of the kinetic energy of the falling rock fragments is dampened by impact; and (c) after the impact the fragment starts rolling down the slope (GLA, 1998).

Under the worst-case scenario, the maximum encroachment of a bouncing rock fragment onto the landfill was estimated to be about 300 feet onto the landfill and the travel time was estimated to be 22 seconds. Based on this analysis, construction of a "catching" wall or other diversion structure near the edge of the landfill is recommended to effectively mitigate the risk of rock fragments rolling onto the landfill. Rockfall trajectories can reasonably be expected to be even shallower and shorter for profiles with gentler slopes. The conclusions reached through the analysis of this profile are of general application throughout the eastern slope of the landfill site.

Debris flows

Earth, mud, and debris flows or debris avalanches form when a mass of unconsolidated sediment is mobilized by sudden ground vibration (e.g., an earthquake) or by a sudden increase in weight and pore water pressure (e.g., after soaking of the soil by heavy rain). Steep topography and deforestation enhance the initial movement of a flow, but once mobilized, flows can spread over gently sloping terrain. Johnson (1970) demonstrated that flows move by "plug flow", in which shear deformation is limited largely to the bottom of the moving mass. The bulk of the mass forms a "plug", in which shear stress is less than the shear strength of the sediment/water mixture. As long as the thickness of the zone of no shear is less than the thickness of the flow, movement will occur, with the plug being passively rafted. If for some reason (e.g., dewatering of the soil mass, or decrease in slope) the thickness of the plug equals that of the flow, movement ceases. This sudden "freezing" of a flow explains the lack of internal stratification, poor sorting of the sediment, and the lobate, steep-sided morphology typical of debris flows.

Review of aerial photographs and geologic mapping of the site, indicate that there is a deposit of poorly-sorted colluvium that could have been formed as a debris flow deposit [Qd(?) on Plate 1; GLA (1998)]. The deposit forms a landform with a rough lobate shape and comparatively steep boundaries, but lacks levees or pressure ridges, and so could also have been formed by erosion of an older colluvial fan.

If not simply the result of erosion of an older colluvial fan, recent studies of the classification of flow type landslides suggest that the deposit identified within Gregory Canyon might be more typical of a debris avalanche (Hungr O., et al., 2001). Hungr et al. (2001) define a debris avalanche as being very similar to a debris flow with the exception that in the case of the debris avalanche, the flow is not confined to an established channel. In the case of a debris flow, the established channel is usually characterized by scour features along a gully path and by the presence of a well-defined depositional cone or fan built up by a number of separate events that follow this same path. In contrast, once the debris avalanche has occurred, it will not normally occur repeatedly at the same location, since depletion of material usually occurs.

It is recognized that the circumstances necessary to form a debris flow/avalanche include a source of loose debris that is typically mobilized by a significant water content, such as a saturated basal layer of material that becomes liquefied by over-riding and rapid loading (Hungr, O. et al, 2001). The susceptibility of the site to these mechanisms was estimated by looking for evidence of previous flow events. Characteristically, a bowl-shaped source area leads to a relatively narrow neck with a U-shaped cross-section. The depositional zone may have multiple lobes of debris overlapping one another, low mounds or levees mark lateral shear surfaces, and arcuate pressure ridges are sometimes seen over the surface of the flow. The deposits themselves are characteristically unstratified and poorly sorted. Sources that might have resulted in the formation of a historical debris avalanche could be the three "hanging" basins that drain the western summit of Gregory Mountain. These basins have gentle slope gradients in their head regions, but drain into the steep western flank of Gregory Mountain. A debris flow/avalanche mobilized within any of these basins would thus have moved rapidly down this steep flank, and accumulate at the toe of the slope, as exemplified, perhaps, by basin 1 and deposit Qd(?) (on Plate 1; GLA, 1998). However, these bowls contain no significant surficial soil deposits.

In siting the landfill within Gregory Canyon, the likelihood of future debris flows/avalanches must be evaluated. Today, most commonly debris flows/avalanches are triggered by a significant increase in saturation, which can buoy the debris down the slope and these types of events typically occur from a rapid snow melt, or in the Pacific

Northwest where heavy rains and flooding are more common. Even if there is an increase in annual rainfall within the vicinity of Gregory Canyon, the climatic conditions are not ideal for the development of repetitive debris flows/avalanches.

The most effective "mitigation" measure to minimize the potential debris flow/avalanche hazard is the natural development of vegetation within the drainage basins. In aerial photographs, these basins generally have a modest to dense development of vegetation along their tributaries and special precautions such as diversion structures near the upper reaches would need to be taken if vegetation is destroyed. The diversion structures should be built so as to be permeable, allowing almost free draining of runoff, but should capture high viscosity earth-, mud- or debris.

3.6 SETTLEMENT ANALYSIS

Refuse settlement in sanitary landfills can be a troublesome and unpredictable problem during the post-closure maintenance period. For example, drainage structures incorporated into the final cover design might become inoperative after a few years because settlement has modified or even reversed the drainage slopes. An analysis of potential settlement is thus a valuable planning tool for maintenance engineering. This section describes the landfill settlement that is estimated to occur in the GCLF during the 30-year period following landfill closure. The settlement analysis provided has been revised from an earlier analysis (GLA, February 1999) and reflects the proposed prescriptive landfill configuration for the 30-year post-closure period.

The mechanics of refuse settlement is complex due to the extreme heterogeneity of refuse fill. According to Edil et al. (1990), the main mechanisms involved in refuse settlement are:

- Mechanical distortion (bending, crushing, and reorientation)
- Raveling (movement of fines into large voids)
- Physical-chemical changes (corrosion, oxidation, and combustion)
- Biochemical decomposition (fermentation and decay)

The magnitude of refuse settlement can thus be inferred to be a function of: (1) the initial refuse density or solid/void ratio, (2) the overall density of the refuse prism or ratio of refuse to daily cover soil, (3) the content of decomposable materials in the refuse, (4) the thickness of refuse lifts and total height of the refuse prism, (5) the refuse prism stress history, (6) the time elapsed since each individual lift was placed, and (7) environmental factors such as moisture content, temperature, and gas content.

In our experience, the most consistent refuse settlement estimates are obtained by modeling the refuse prism as a 3-dimensional net, calculating the settlement at each node of the net with a time-dependant exponential decay function, and adding the total settlement for each column of the net at selected time intervals. Based on the work of Huitric (1981), settlement was modeled as an exponential decay function of the form:

$$\text{Remaining settlement} = \mathbf{a}Te^{-bt}$$

where **a** and **b** are constants such that total expected settlement is a proportion (**a**) of the original thickness, (T), of a particular lift of refuse, and settlement occurs at an exponential rate of **bt**, where t is the time (in years elapsed), since the particular lift of refuse was placed. For a municipal solid waste landfill with standard compaction characteristics, **a** varies between 0.2 and 0.35, and **b** varies between 0.10 and 0.11. Since the GCLF master plan calls for a fairly high compaction ratio, a value of 0.25 was used for parameter **a**, and an intermediate value of 0.105 was used for parameter **b** (Figure 3-8).

In estimating post-closure settlement, a two-dimensional grid over the footprint of the refuse prism was established, with a nodal spacing of 250 feet (Figure 3-9; Table 1). The third dimension in the model net was then added to represent the change in elevation between discrete time intervals. As currently constructed, each "layer" of the model represents 3 to 4 years of landfilling (Table 3-1).

Figure 3-10 shows the landfill surface elevations at the time of landfill closure (approximately year 30), and Figures 3-11a through -11c depict the estimated landfill elevations 10, 20 and 30 years after closure, respectively. The calculated values for the nodes used to contour these figures are summarized in Table 3-2.

As shown on Figure 3-11c, total potential settlement during a 30-year post-closure period could be as much as 60 feet in the southern half of the landfill prism. This is less than the 140 feet estimated in the earlier settlement analysis (GLA, February 1999) for two reasons. First, the thickness of refuse has been reduced as the floor of the landfill has been raised; and second, a higher initial compaction has been assumed. In addition, the configuration of the refuse prism changes slightly with the "pyramid" that formed the apex eventually settling into a "deck". Even after 30 years, if proper maintenance is performed, surface grades should be sufficient to facilitate efficient drainage off the site.

3.7 LEACHATE GENERATION ANALYSIS

This section complements GLA's earlier leachate generation analysis report dated December 18, 1998 and includes a discussion of the results following extension of the model from a 20-year post-closure period to a 30-year post-closure period.

Modeling of potential leachate generation was performed using the United States Army Corps of Engineers HELP 3 (Hydrologic Evaluation of Landfill Performance) computer program, which uses representative rainfall and evapo-transpiration data to determine the amounts of leachate that might be generated in municipal solid waste landfills. The program takes into account the total area landfilled, representative precipitation patterns, representative evapo-transpiration, and the hydraulic conductivity of various construction materials to calculate leachate generation and accumulation.

<u>Climate.</u> The initial climate properties were selected from a table of default values included in the HELP 3 model. These default values were selected for the city of San Diego and were corrected for the latitude of the proposed Gregory Canyon Landfill.

Precipitation data were adjusted to a 60-year annual average of 19.3 inches, with a minimum yearly total of 8.36 inches and a maximum yearly total of 34.8 inches (Figure 3-12). In contrast, in the model provided earlier (GLA, 1998), the precipitation data were adjusted to a 50-year annual average of 18 inches, with a minimum yearly total of 4.40 inches and a maximum yearly total of 24.79 inches. The slightly different values arose when a new synthetic precipitation record was generated for this analysis.

Material properties. The engineering and hydraulic properties of materials were determined from HELP3 default values, as follows (from top to bottom):

Layer	USCS	Thickness (inches)	Porosity	Saturated Hydraulic Conductivity (cm/sec)
Vegetative cover	SM	18	0.473	5.20E-04
60 mil LLDPE	--		--	2.00E-13
Foundation layer	SM	24	0.473	5.20E-04
Refuse	--	Variable	0.671	1.00E-03
Operations layer	SM	24	0.473	5.20E-04
LCRS layer	GP	24	0.397	3.00E-02
60 mil HDPE	--		--	2.00E-13
Low permeability layer	CH	24	0.4	7.80E-07

In calculating the potential percolation through the first (upper) liner HDPE and the final cover LLDPE geomembranes, densities of two pinholes per acre and one installation defect per acre were assumed. The hydraulic conductivities and other engineering properties of materials utilized in the analysis are shown in the HELP3 output sheets presented in Attachment 6.

Model configuration. Modeling of the leachate generation for the Gregory Canyon Landfill was performed by subdividing the 186-acre landfill (the approximate area of the landfill excluding the three transmission line pads on the east side of the landfill) into three zones. The first zone encompasses the two portions of the "floor" area that have a slope of 3% (24.6 acres); the second encompasses the intervening portion of the "floor" area that has a slope of 8.5% (16.0 acres); and the third zone encompasses the "slope" areas (145.8 acres).

Leachate drains in the LCRS system were positioned at 500-foot intervals within the bottom LCRS gravel. Along the side slopes, drains were assumed to be positioned at each construction bench (i.e., 100-foot horizontal spacing).

Closure of the proposed Gregory Canyon Landfill was modeled using a prescriptive final cover (CCR Title 27) including a 24-inch foundation layer, a 60-mil linear LLDPE geomembrane, and an upper 24-inch topsoil layer able to support vegetation.

Modeling period. Using the climatic and material values described above, the HELP 3 analysis was performed iteratively for a simulated 31-year post-closure period. Following the practice described by Peyton and Schroeder (1988), the first year (year 30) is used to initialize the model and its results are not reported as part of the simulation.

Results. Based on the HELP3 model data, the total leachate generation and peak daily leachate generation during the 30-year operating life of the landfill is generally low, except for significant "spikes" associated with the heavy precipitation in years 3, 16 and 22 (Figure 3-12). After final cover is placed at the end of year 30, leachate generation becomes very low. During the landfill's operating life, the amount of leachate generated reaches a maximum value in the 16th year, when the projected total leachate generated is estimated at 53,984 ft^3 (403,800 gallons), of which 8,187 ft^3 (61,239 gallons) are generated from the floor area and 45,797 ft^3 (342,561 gallons) are generated from the slope area. The peak daily leachate generation is estimated to be 142 ft^3 (1,062 gallons) for the floor area and 1,094 ft^3 (8,183 gallons) for the slope areas during the 16th year. The peak daily head on the liner reaches 0.25 inches during the 16th year.

For the 30-year post-closure period, the amount of leachate generated peaks 29 year after closure, when the projected total leachate generation is estimated to be 3,381 ft^3 (25,290 gallons) of which 735 ft^3 (5,498 gallons) are generated in floor areas and 2,646 ft^3 (19,792 gallons) are generated is slope areas. The peak daily leachate generation is estimated to be 2.5 ft^3 (19 gallons) in floor areas and 9.7 ft^3 (73 gallons) in slope areas 26 year after closure.

During the post-closure period, the calculated peak daily head on the liner reaches a maximum of 0.03 inches. The peak daily head during the 30-year operating life of the landfill (0.25 inches) and during the 30-year post-closure period (0.03 inches) are well within the 12-inch range allowed by regulations (e.g., 40 CFR, Subpart D, Section 258.40).

The results of the three simulations performed are included in Attachment 6. The results from GREG3FL simulation represent the portions of the floor that have an average floor grade of 3%. The results from GREG3ST simulation represent the portions of the floor that have an average floor grade of 8.5%. Finally, results of GREG3BS simulation represent the back slopes with an average grade of 50%.

3.8 CONCLUSIONS AND RECOMMENDATIONS

Based upon the geotechnical analyses completed in support of the proposed GCLF design, it is concluded that the landfill meets or exceeds all federal and state design criteria. The following findings support this conclusion:

Stability Analyses

- Analysis of the dominant directions of discontinuities in the bedrock within Gregory Canyon indicates that it is not feasible for substantial large-scale, block-slip movement or wedge failure to occur in cut slopes excavated to a maximum gradient of 2:1. It is recommended that cut slopes be mapped as the excavation proceeds and this conclusion be reevaluated during the excavation process. Based on the nature of the bedrock within Gregory Canyon, it is possible that localized, small-scale block falls could occur.

- In addition, because the excavated areas of the site will be underlying by cohesive bedrock, circular failure in the cut slopes is unlikely.

- For the borrow/stockpile areas A and B, stability analyses indicated that based on the proposed design, the stockpiles are considered to have adequate stability.

- Static stability analyses were performed for the GCLF refuse prism along cross-section A-A' extending down the center of the canyon. The results of the stability analysis, using the computer program SLOPE/W (Geo-Slope, 1995), indicate that the static factor of safety for this configuration is calculated to be 1.50, and meets CCR 27 standards.

- Dynamic stability analyses were performed to assess the behavior of the refuse prism under seismic loading. Initially, pseudo-static stability analysis was performed, however, because the refuse prism did not meet the required factor of safety of 1.5, a more rigorous seismic-induced permanent displacement analysis was performed. Results of this analysis employed methods developed by Bray and Rathje (1998) and calculated a displacement of less than one inch under the maximum credible earthquake (MCE) site acceleration of 0.40g. This estimated displacement is less than the currently accepted standard of practice for liner systems of 6 to 12 inches (Seed and Bonaparte, 1992).

- Stability analyses were performed for the proposed final cover system for closure of the GCLF. As with the refuse prism, initially pseudo-static stability analysis was performed, however, because the pseudo-static factor of safety was less than 1.5, a more rigorous additional analysis was performed to estimate the seismic induced permanent displacement during the postulated MCE event at the site. Results of this more rigorous analysis employed methods developed by Makdisi and Seed (1978) and Bray and Rathje (1998) and calculated displacement of less than one inch to 5.1 inches under the maximum credible earthquake (MCE) site acceleration of 0.40g. These estimated displacements are less than the currently accepted standard of practice of 6 to 12 inches and would not be expected to inhibit the functional integrity of the cover.

Liquefaction Susceptibility

- GLA evaluated the liquefaction susceptibility of the loose alluvial sediment at the toe of Gregory Canyon. The results of this analysis indicate that under existing conditions, factors of safety ranging from 1.30 to 3.34. As a results the liquefaction susceptibility of these alluvial sediments are low.

Rockfalls and Debris Flows

- An assessment of rockfall trajectories was completed for the GCLF, using the steepest (eastern) slopes at the site. Under bouncing trajectories calculated assuming some dampening of the kinetic energy by impact, it is calculated that the bouncing particle (rock) will stop within a few feet from the limit of refuse with a travel time of about 23 seconds.

- Another scenario including an assumption that the rock fragment will start to roll down the slope, results in rockfall extending as much as 360 feet onto the landfill.

- Construction of a "catching" wall, or diversion structure is recommended to mitigate the risk of rockfall.

- GLA mapped a lobate shaped, poorly sorted colluvium deposit, which might have formed from a debris flow, or may have formed by erosion of an older colluvial fan. As a conservative measure, an assessment of the occurrence of debris flows was conducted. The likelihood of a debris flow encroaching on the landfill is considered low considering current and forecast climatic conditions in the area.

Settlement Analysis

- A settlement analysis was performed for the proposed GCLF for the 30-year post-closure maintenance period. Results of this analysis indicate a maximum of 60 feet of post-closure settlement in the southern half (thickest portion) of the landfill prism.

- Results of the settlement analysis indicate that although the refuse prism will experience significant settlement, the grades generally appear to remain adequate for site drainage.

Leachate Generation

- During the 30-year operating life of the landfill, leachate generation is generally low, with the exception of significant "spikes" associated with heavy rain in years 3, 16 and 22. Leachate generation becomes even lower following closure of the landfill and placement of the final cover at the end of year 30.

- The model results indicate that the largest amount of leachate generation occurs in year 16 when the projected total leachate generated is estimated at 53,984 ft^3 (403,800 gallons), peak daily leachate generation is estimated to be 1236 ft^3 (9,245 gallons), and the peak daily head on the liner reaches 0.25 inches.

- Following landfill closure, the amount of leachate generated peaks in the 29^{th} year after closure, when the projected total leachate generation is estimated at 3,381 ft^3 (25,290 gallons). Peak daily leachate generation is estimated to be 12.2 ft^3 (92 gallons), 26 years after closure and the peak daily head on the liner over the post-closure period is calculated to be 0.03 inches.

3.9 ADDITIONAL GEOTECHNICAL REFERENCES

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